PUMPED STORAGE HYDROELECTRIC POWER PLANT STUDY, OAHU, HAWAII

OCEAN ENGINEERING CONSIDERATIONS FOR THE INLET/OUTLET STRUCTURE AT THE KOKO CRATER SITE

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December 1993

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1.0 INTRODUCTION

Hawaiian Electric Company has performed a reconnaissance level study identifying the potential feasibility of a pumped storage hydroelectric power plant at two sites on Oahu: Koko Head Crater (which uses sea water) and Ka'au Crater (which uses fresh water). As a result of this work and the desire of the State of Hawaii to further explore the feasibility of these projects and to select the more feasible project for subsequent consideration, the Department of Land and Natural Resources (DLNR) has contracted with Okahara & Associates, Inc. to undertake a prefeasibility study to provide more accurate estimates of developing each site and its potential. The study would give more specific indication of the technical feasibility of the sites and potential environmental impacts.

Edward K. Noda and Associates, Inc. was retained to provide conceptual ocean engineering criteria and considerations related to the ocean inlet/outlet structure for the Koko Crater facility site. This report generally describes the physical oceanographic environment at the proposed inlet/outlet structure location, design considerations affecting alternative inlet/outlet structure concepts, and potential oceanographic impacts related to construction and operation.

2.0 PHYSICAL OCEANOGRAPHIC ENVIRONMENT

Based on available information, this section summarizes the physical oceanographic environment at the proposed site for the inlet/outlet structure associated with the pumped storage hydroelectric plant at the Koko Crater site. The primary factors include bathymetry, the littoral processes (typical waves and currents), and potential storm wave impacts.

The Koko Crater site is located on the southeast end of Oahu between Koko Head and Makapuu Point. Figure 1 shows the location and the sectors of wave exposure for the site. Figure 2 shows a vicinity map and topographic features at the site. Two specific locations are being considered for the inlet/outlet (Site A and Site B), depending on the construction options as described in Section 3.0.

The island mass shelters the site from winter North Pacific swell. These waves undergo considerable diffraction and refraction effects prior to reaching the site as much reduced wave heights. The site is directly exposed to the predominant northeast tradewind waves and to summer southern swell. Normally a high wave energy environment during the summer months when the tradewinds are persistent and strong, the site is calmest during the winter months when the trades weaken and winds can be light and variable. However, infrequent Kona storm waves from the southwestern quadrant can impact the site during winter months. Infrequent hurricanes passing south of the islands (traveling from the southeast to southwest direction) also generate sizeable waves that can impact the site.

Wave data from a Waverider buoy situated offshore Makapuu Point[1] is the most representative long-term data to describe the typical offshore wave climate at the site. The Waverider buoy is moored in about 400-foot water depth offshore Makapuu Point, and is more exposed to the winter North Pacific swell than the project site location. Therefore, while the wave data during winter months over-estimates the wave conditions at the project site, the data for the summer months can be considered applicable to the project site. Table 1 summarizes the wave data obtained over an eight year period. Percent frequency of occurrence of significant wave height versus wave period is provided for the summer season (May-Oct) and winter season (Nov-Apr). An annual summary is also provided for 1988 (representing a typical year and one in which there were no data gaps in the record). During a typical year, the data indicates that waves are less than 8 feet the majority of the time, with periods generally less than 8 seconds.

From the existing data, the water depth at the proposed shoreline site for the inlet/outlet structure is relatively deep near the base of the shoreline cliff, estimated to be approximately 30-40 feet below MLLW. From the NOAA hydrographic chart of the vicinity (Figure 3), the nearshore bottom slope is approximately 1V:13H from the base of the shoreline cliff to 60-foot water depth about 400 feet from shore. Because of the relatively deep nearshore depths, the predominant tradewind waves undergo little refraction effects and can approach at oblique angles to the shoreline.

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[1] Coastal Data Information Program, sponsored by the U.S. Army Corps of Engineers, data reports by the Scripps Institution of Oceanography.

TABLE 1
MEASURED WAVE DATA OFFSHORE MAKAPUU POINT

% Frequency Occurrence of Significant Wave Height vs. Period

	Hs/Ts	4-6	6-8	8-10	10-12	12-14	14-16	16-18	TOT%
Summer	<2'								0.0
1981-	2-4'	2.9	6.1	0.5	0.2				9.7
1988	4-6'	25.5	27.0	1.8	0.7	0.4	0.1		55.7
	6-8'	4.1	21.9	1.5	0.8	0.1			28.5
	8-10'	0.1	4.1	0.5	0.5	0.1			5.4
	10-12'	0.1	0.3	0.3					0.8
	TOT%	32.6	59.5	4.5	2.4	0.8	0.2		100
Winter	<2'								0.0
1981-	2-4'	0.4	3.2	0.5	0.2				4.5
1988	4-6'	7.4	15.7	4.6	3.5	0.9	0.2		32.2
	6-8'	5.1	21.6	5.0	3.6	1.3	0.2		36.9
	8-10'	0.5	11.7	2.3	1.8	1.1	0.3		17.8
	10-12'		3.2	0.8	0.8	0.3			5.2
	12-14'		1.2	0.6	0.2	0.1	0.1		2.4
	14-16'		0.2	0.3	0.2				0.8
	16-18'								0.2
	TOT%	13.5	57.0	14.2	10.5	3.7	0.9	0.1	100
Annual	<2'								0.0
1988	2-4'	1.2	3.9	0.2	0.1				5.5
	4-6'	19.5	22.3	2.2	1.6	0.1			45.7
	6-8'	6.1	24.0	3.2	2.4	0.5	0.1	0.1	36.5
	8-10'	0.5	6.1	0.9	1.1	0.8	0.2		9.7
	10-12'		0.8	0.3	0.5	0.2			1.7
	12-14'		0.2	0.2			0.2		0.6
	14-16'		0.1	0.1	0.1				0.3
	16-18'					0.1	0.1		0.2
	TOT%	27.3	57.5	7.1	5.9	1.6	0.6	0.1	100

Hs = significant wave height
Ts = significant wave period

The nearshore currents are relatively strong and persistent. Figure 4 shows the circulation patterns and currents in the vicinity of the site.[2] Flood tide currents set alongshore in the southwestward direction (towards Koko Head). Ebb tide currents set offshore in the east-northeastward direction. Current data obtained approximately 1.3 miles offshore the site indicate that there is a consistent overall net drift in the southwestward direction (flood tide currents are more persistent and stronger than ebb tide currents). Maximum measured flood tide current was about 1.2 knots, while maximum measured ebb tide current was about 1 knot.

The coastal reach at the proposed site of the inlet/outlet is a rocky, wave swept shoreline. There is little sediment along this coastal cliff site. Sandy Beach Park is situated approximately 1 mile northeast of the project site, and Hanauma Bay is situated approximately 1 mile southwest of the project site. Halona Blowhole (a visitor attraction) is located approximately 1,500 feet northeast of the site, around a rocky point and on the opposite side of Halona Cove. Figure 5 is a reference map showing points of interest along this coastal reach.[3] The rocky point just northeast of the project site is shown to be the site of the Honolulu Japanese Casting Club Monument. This rocky point is apparently a popular fishing spot.

Because of the wave exposure and relatively deep water depths near the shore, the site is vulnerable to large storm wave

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[2]From "Circulation Atlas for Oahu, Hawaii", by Karl H. Bathen, published by the University of Hawaii Sea Grant College Program, Sea Grant Miscellaneous Report UNIHI-SEAGRANT-MR-78-05, April 1978.

[3]From "Reference Maps of the Islands of Hawai'i, Fourth Edition, Full Color Topographic Map of O'ahu", published by University of Hawaii Press.

activity. Deepwater hurricane-generated waves and Kona Storm waves can impact the site with large breaking wave heights at the shoreline. Assuming a water depth of 30 feet near the base of the shoreline cliff, deepwater design wave height of 30 feet[4] with 12 second period, and a bottom slope of 1V:13H, the design breaking wave height is about 38 feet and the depth at which the design wave initiates breaking is also about 38 feet.[5]

[4]From "Hurricane Vulnerability Study for Honolulu, Hawaii, and Vicinity, Volume 2, Determination of Coastal Inundation Limits for Southern Oahu from Barbers Point to Koko Head", prepared for the U.S. Army Engineer Division, Pacific Ocean, prepared by Charles L. Bretschneider and Edward K. Noda and Associates, Final Report dated May 1985. Estimated deepwater design wave based on SE Model Scenario Hurricane, wave approach direction from approximately 175 degrees true.

[5]Breaking wave height and breaking depth as determined from the "Shore Protection Manual", U.S. Army Corps of Engineers, Coastal Engineering Research Center, Waterways Experiment Station, 1984.

3.0 DESIGN CONSIDERATIONS AFFECTING THE INLET/OUTLET

Figure 6 depicts the conceptual plan for conveying water to and from the ocean and Koko Crater. The required tunnel size between the powerhouse and the inlet/outlet structure is 25 feet. Two basic alternatives are available for the inlet/outlet structure. These are: (1) continuous tunneling offshore to the inlet/outlet structure location; (2) tunneling to the shoreline, with a conduit extending to the base of the cliff and an offshore breakwater to protect the inlet/outlet. Figure 7 shows these two options.

Site A is the preferred location for the first option (Option A, Figure 7-a), because tunneling distance between Koko Crater and the inlet/outlet location is minimized. The inlet/outlet structure would be extended sufficiently far offshore such that it would not be subject to large breaking waves. For an estimated deepwater design wave of 30 feet with 12-second period, the breaking depth is about 38 feet. Therefore, based on the estimated bathymetry along the tunnel alignment, it is recommended that the inlet/outlet structure be located at least about 300+ feet from shore in water depth of about 50 feet or greater. While the inlet/outlet structure would not be subjected to breaking wave forces, the structure would still need to be designed for stability under the wave velocities and accelerations imposed by the design wave conditions. As depicted in Figure 7-a, the inlet/outlet is extended about 500 feet offshore to water depth of about 65 feet, at which point the conduit is fully exposed on the ocean bottom with a clearance depth above the conduit of about 30 feet.

Site B is the preferred location for the second option (Option B, Figure 7-b). This option requires the initial construction of a cofferdam so that the conduit could be constructed in the "dry".

An offshore breakwater is also necessary to provide wave protection. The shoreline configuration at Site B is ideal because the small cove can be enclosed more easily by the breakwater. Because of the relatively deep water depths near the shoreline, the conduit need not be extended a great distance offshore to reach sufficient depth of water for the inlet/outlet. It is estimated that a conduit length of less than 100 feet may be required, dictated primarily by the requirement to place the breakwater at least about 150 feet from shore to provide sufficient work area. As depicted in Figure 7-b, the inlet/outlet daylights at the base of the shoreline cliff, with excavation of the ocean bottom to the invert depth of about 50 feet. The breakwater provides a wave-protected environment for the inlet/outlet during construction and operation. Because large breaking waves could be expected at the shoreline, the breakwater structure would protect the inlet/outlet from storm wave impact and prevent large fluctuations in the water surface elevation.

The breakwater structure would preferably be a rubblemound structure. The rubblemound breakwater would not only dissipate wave energy more effectively than an impervious structure, but would also serve to "filter" large objects from the intake waters. For a rubblemound breakwater structure, the armor size would necessarily have to be very large for stability under the design wave conditions. Assuming the use of dolos concrete armor units, the individual dolos units would be on the order of 40 tons. Figure 8 shows a conceptual typical section for a rubblemound breakwater. The conceptual design was developed using the Automated Coastal Engineering System (ACES)[6] computer

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[6]Automated Coastal Engineering System (ACES) Version 1.07a, April 1993, developed by the Automated Coastal Engineering Group, Research Division, Coastal Engineering Research Center, U.S. Army Engineer Waterways Experiment Station.

program. The application for breakwater design provides estimates for armor weight, minimum crest width, armor thickness, and the number of armor units per unit area of a breakwater using Hudson and related equations.

The breakwater crest elevation need not be high enough to prevent wave overtopping during design wave conditions. The primary consideration is to reduce wave heights sufficiently to permit construction and efficient operation of the inlet/outlet. For the conceptual breakwater design, the transmitted wave heights were estimated using the ACES computer program. The ACES application for determining wave transmission through a permeable structure uses a method developed for predicting wave transmission by overtopping coefficients using the ratio of breakwater freeboard to wave runup (suggested by Cross and Sollitt, 1971), combined with the model of wave reflection and wave transmission through permeable structures of Madsen and White (1976). Table 2 provides the results for a range of wave conditions.

A breakwater crest elevation of +12 feet MLLW would result in minimal or no wave overtopping during typical high wave conditions (say up to 18-foot waves that could be expected on an annual basis). However, because of the permeability of the structure, transmitted wave heights would be about 3 feet (or less). The transmitted wave height for the design wave condition would be about 7 feet due to both overtopping and transmission through the structure. The breakwater crest width and crest elevation are considered the minimum necessary. A higher or wider crest would result in reduced wave transmission, but with greater cost and visual impacts.

TABLE 2
TRANSMITTED WAVE HEIGHTS FOR BREAKWATER
UNDER VARIOUS WAVE CONDITIONS

Wave Conditions	K_R	K_{Tt}	K_{To}	K_T	H_T (ft)
10 ft, 14 sec south swell	0.57	0.19	0.0	0.19	1.9
14 ft, 14 sec extreme south swell	0.57	0.16	0.04	0.17	2.3
14 ft, 9 sec storm-generated waves	0.18	0.15	0.0	0.15	2.1
18 ft, 10 sec storm-generated waves	0.23	0.14	0.07	0.16	2.8
22 ft, 11 sec storm-generated waves	0.33	0.13	0.14	0.19	4.2
26 ft, 11 sec storm-generated waves	0.32	0.12	0.18	0.22	5.6
30 ft, 12 sec design wave	0.42	0.11	0.22	0.24	7.2

K_R = wave reflection coefficient
K_{Tt} = wave transmission coefficient through structure
K_{To} = wave transmission coefficient by overtopping
K_T = total wave transmission coefficient = $(K_{Tt}^2 + K_{To}^2)^{1/2}$
H_T = transmitted wave height = K_T x incident wave height

Other design alternatives are available for the breakwater structure, such as using concrete caissons or other concrete wave-absorbing structures. These structures would be pre-fabricated and installed in modules to form the continuous breakwater. Generally, such concrete structures are more costly to construct than a rubblemound structure. If designed to permit throughflow, they are also more difficult to design with respect to wave energy absorption and wave transmission characteristics. However, depending on the availability of materials for the rubblemound structure and the constructability aspects (wave exposure and accessibility), modular concrete breakwater alternatives may be cost-competitive.

4.0 POTENTIAL OCEANOGRAPHIC IMPACTS

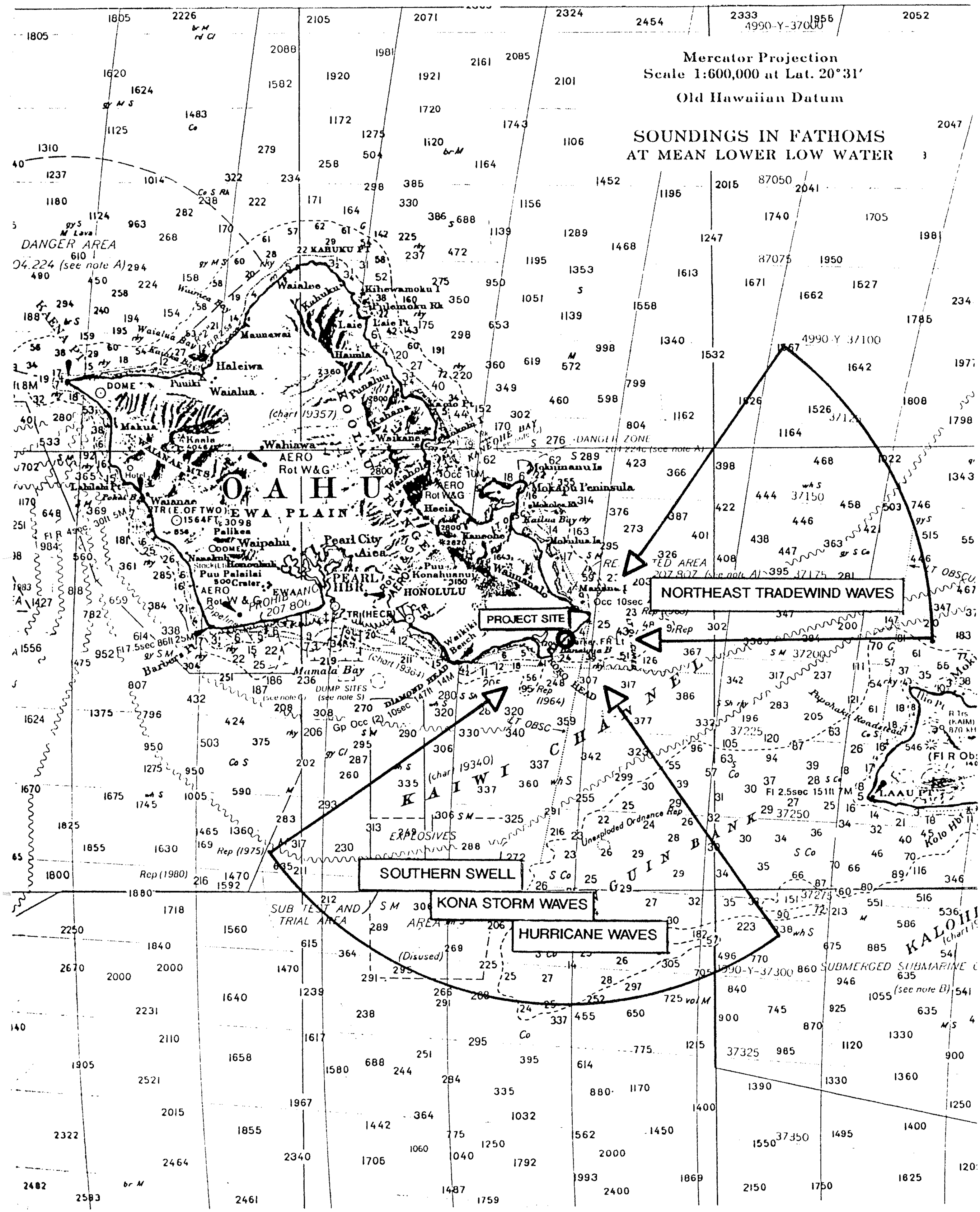
The potential significant oceanographic impacts during construction are primarily related to turbidity generated by the in-water activities and the area of ocean bottom impacted by the construction. The continuous tunneling option would result in the least impacts since the in-water activities would be limited in scope and duration. Disturbance to the ocean bottom would occur only along the tunnel alignment after it daylights at the ocean bottom. Because of the deep depths, wave exposure, and strong currents, silt-containing devices (such as silt screens) would not be effective. However, the turbidity impacts would be expected to be minimal since the high energy ocean environment would quickly disperse the silts that may be generated by the excavation.

The second option, where the conduit daylights at the shoreline cliff and is protected by an offshore breakwater, would not generate significant turbidity if the construction is performed in the dry. However, construction of the rubblemound breakwater could result in turbidity generated over a more extended time frame, but with lower turbidity levels than associated with breaking through the ocean bottom (which may require the use of explosives). The cofferdam construction, to enable the installation of the conduit in the dry, would impact the shoreline area because the water areas landward of the cofferdam would be filled after installation of the conduit. The rubblemound breakwater, while permanently covering the ocean bottom under its footprint, would be expected to enhance the marine biota in the vicinity by providing a more diverse habitat. In addition to the new tidal and subtidal habitat created by the breakwater slopes, the protected waters within the confines of the breakwater would also provide sheltered habitat where none currently exists along this wave-exposed shoreline.

Neither options would significantly impact existing littoral processes. Because of the paucity of sand in the offshore area, potential impacts to littoral transport is not an issue. There would be no impacts to the sandy beach areas located about 1 mile northeast of the site nor to Hanauma Bay located approximately 1 mile southwest of the site.

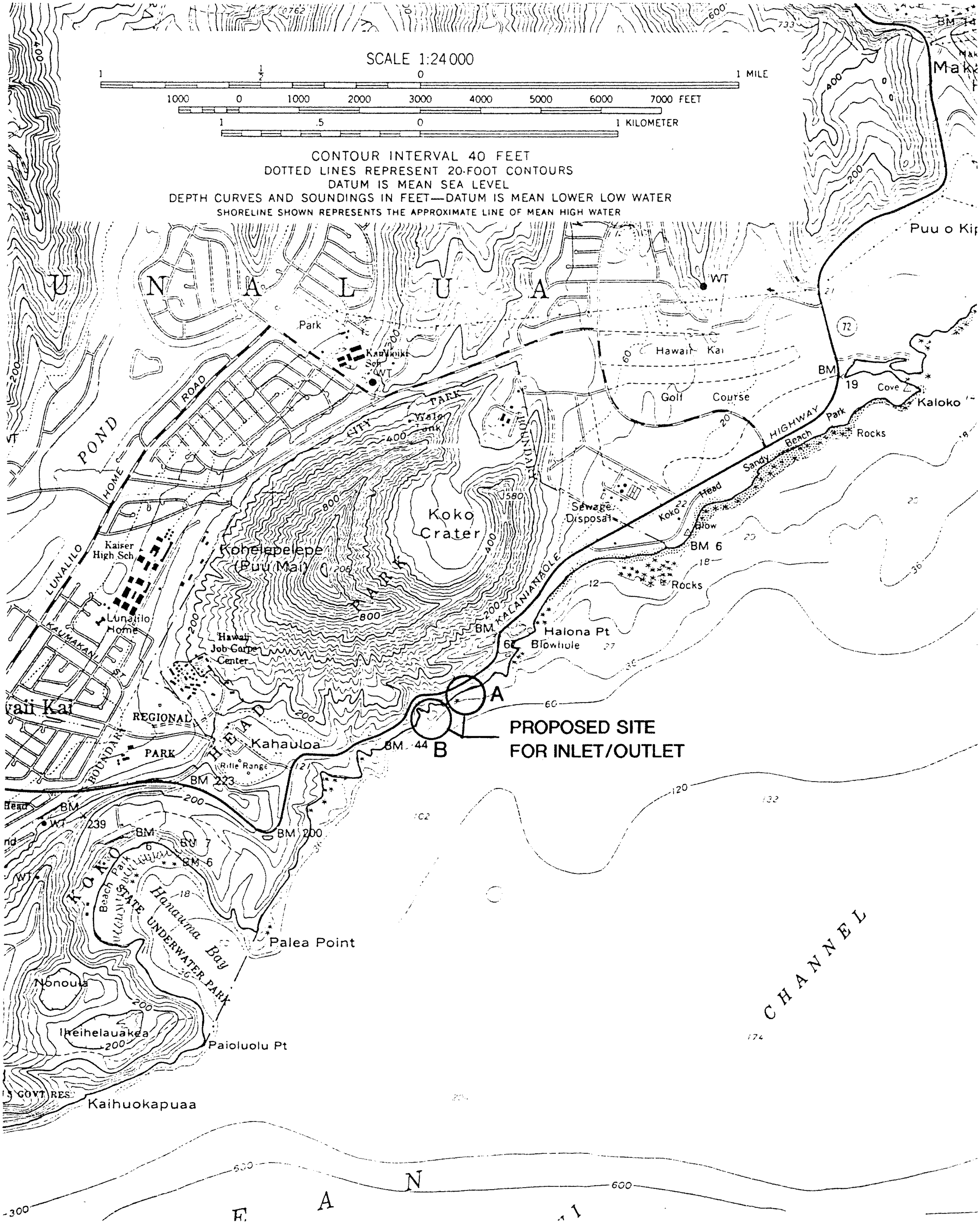
The project site is also sufficiently isolated from Halona Blowhole, such that there will be no significant impacts in the short-term or long-term due to the in-water construction.

There are potential public safety concerns due to the nearshore or offshore structures. For the offshore inlet/outlet structure, there is a concern with respect to the safety of divers who may be "caught" by the high flows. The inlet/outlet should be designed to prevent divers (or other large marine animals) from either approaching too close to the inlet/outlet (i.e. provide a cage structure around the inlet/outlet), or from being entrained by the flows (i.e. by design of the inlet/outlet structure). For the breakwater-enclosed inlet/outlet option, the shoreline should be adequately secured to prevent access to the breakwater-enclosed water area. Because there is always the possibility that persons may trespass into the secured shoreline area, the inlet/outlet should also have measures to prevent entrainment by intake flows.



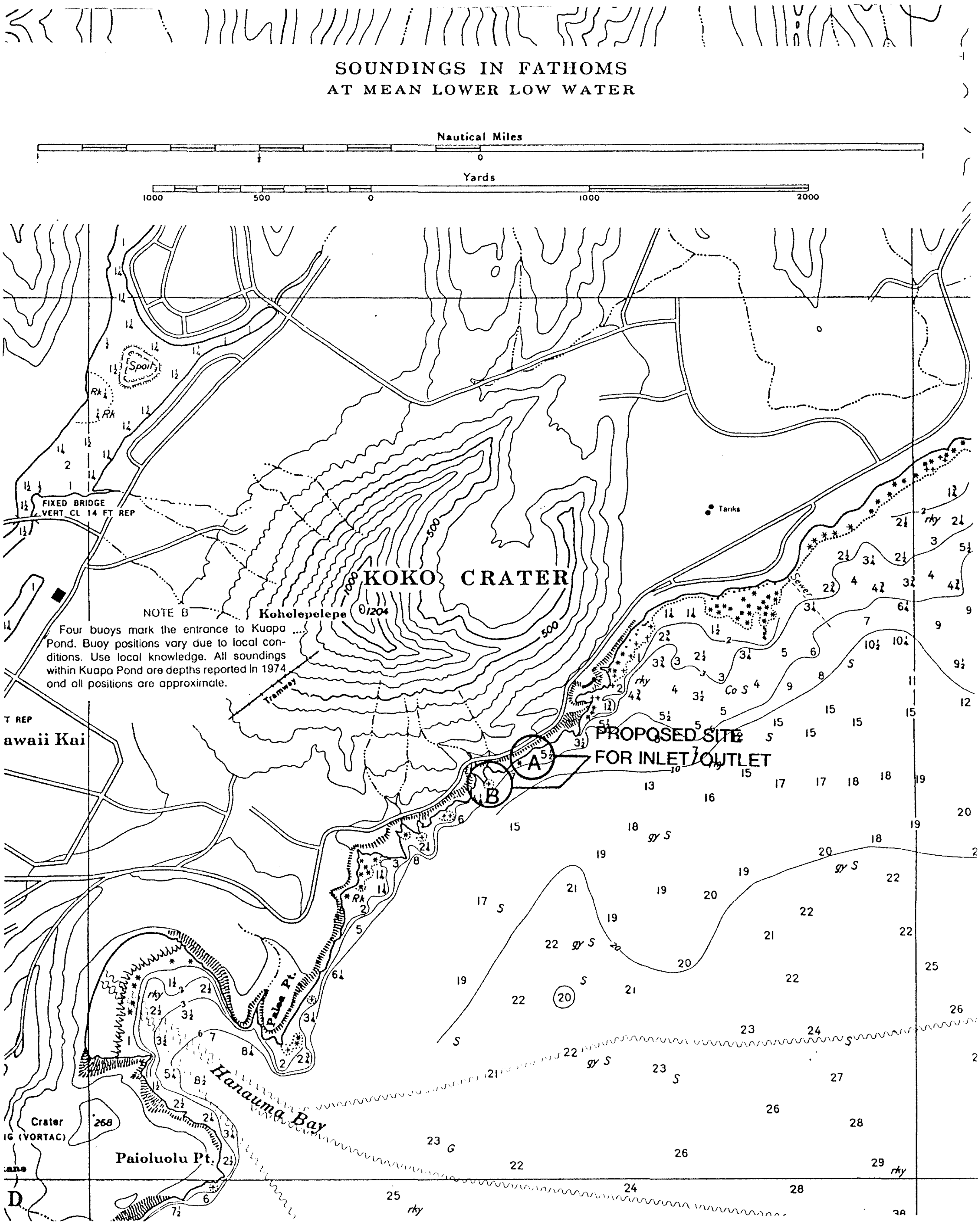
Location Map Showing Sectors of Wave Exposure

FIGURE 1



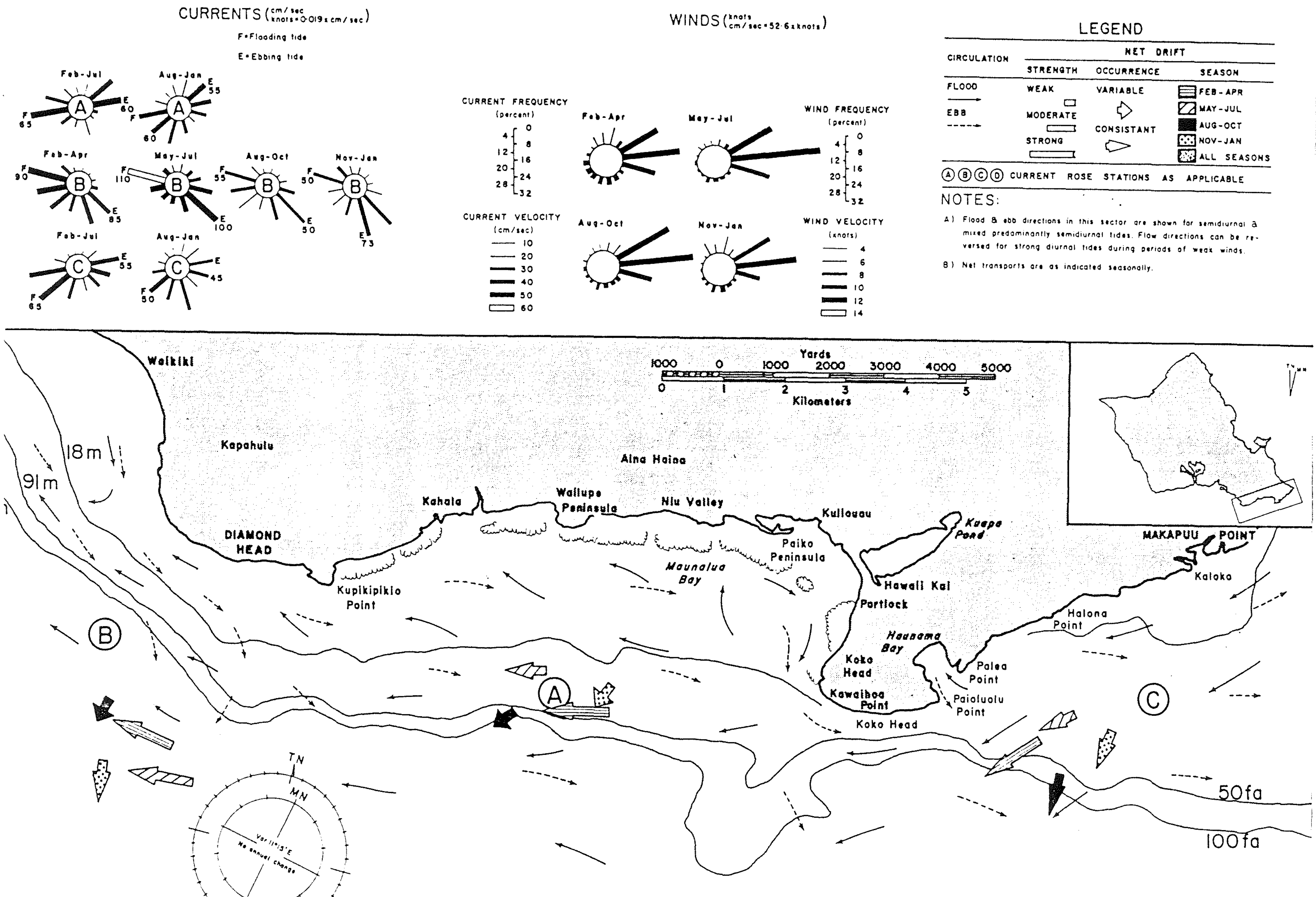
USGS Topographic Map of Vicinity (Scale 1:24,000)

FIGURE 2



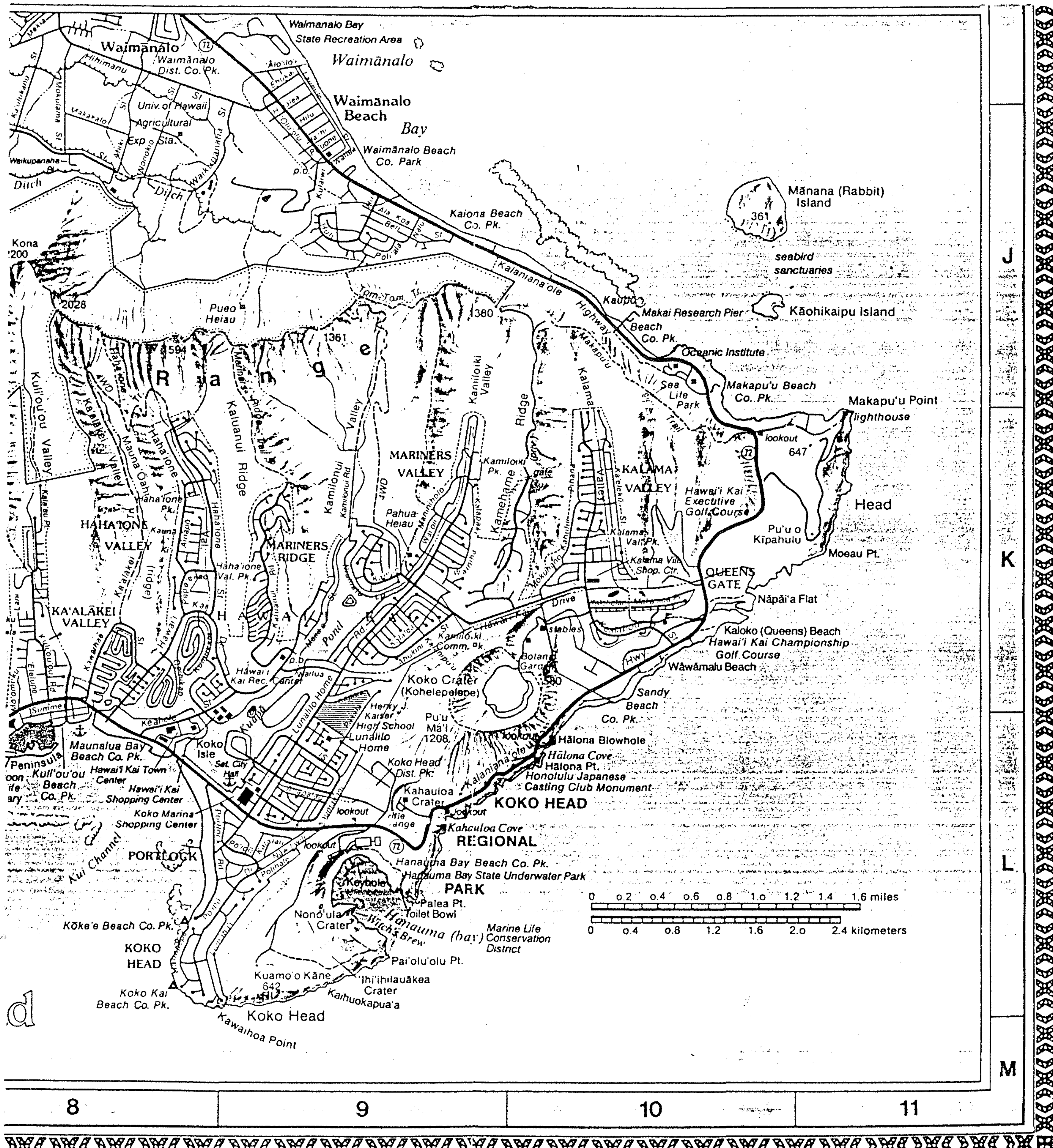
NOAA Hydrographic Chart of Vicinity (Scale 1:20,000)

FIGURE 3



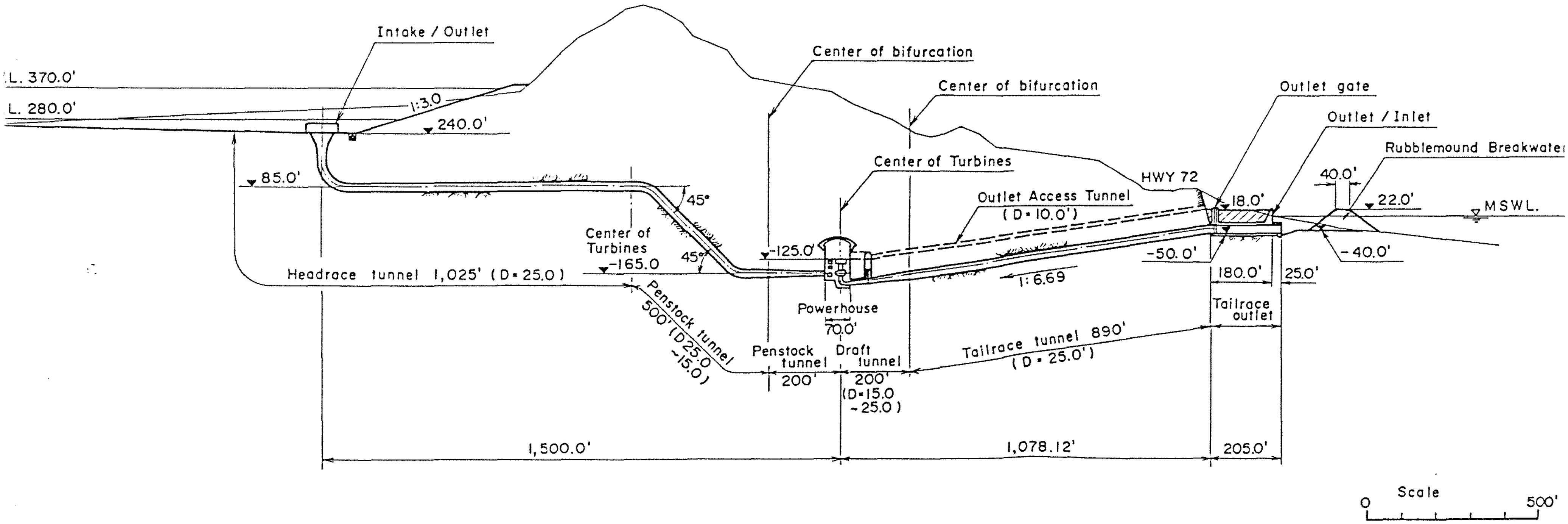
Nearshore Currents and Circulation Patterns in Vicinity
(from Circulation Atlas for Oahu, Hawaii by Karl H. Bathen, 1978)

FIGURE 4



Reference Map Showing Points of Interest
(published by University of Hawaii Press)

FIGURE 5



Conceptual Profile for Koko Crater Pumped Storage Hydroelectric System
(from EPDC International Ltd.)

FIGURE 6

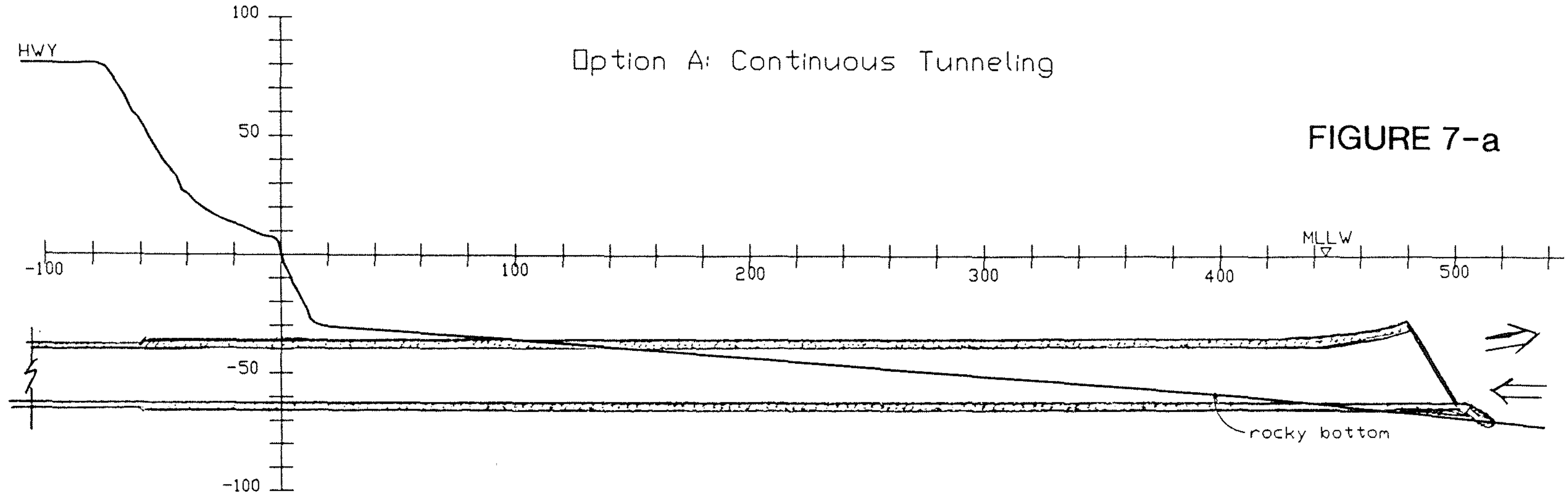


FIGURE 7-a

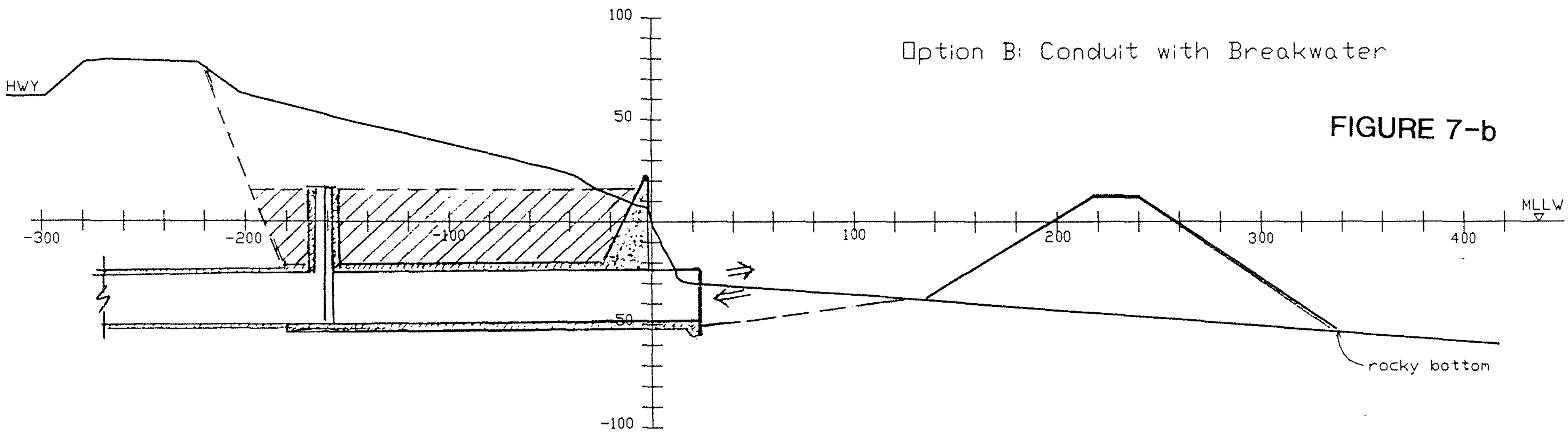
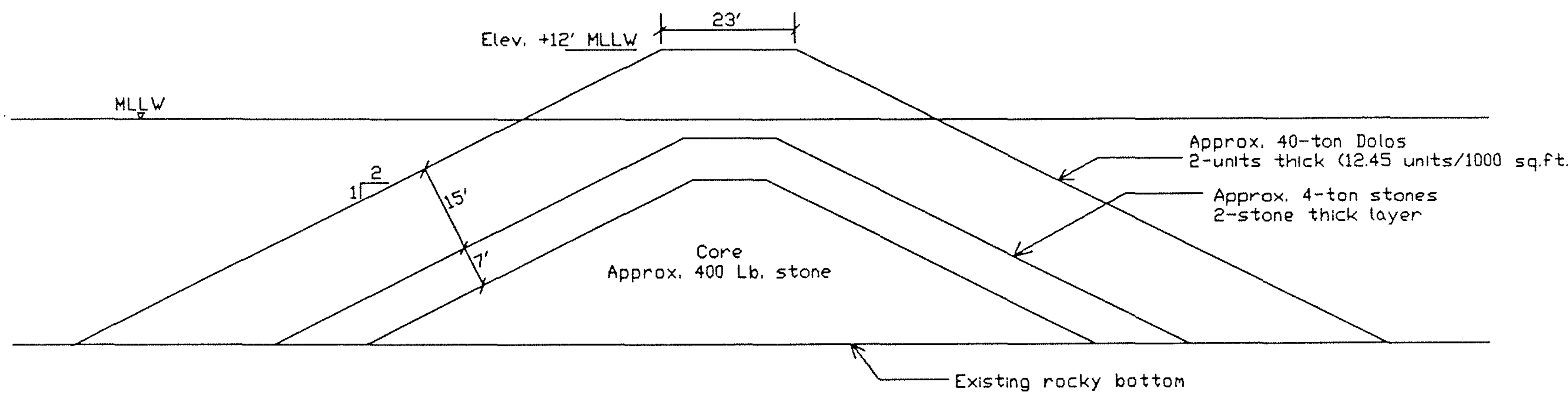


FIGURE 7-b

Conceptual Profile for Inlet/Outlet Options
(after EPDC International Ltd.)

FIGURE 7



TYPICAL BREAKWATER SECTION

Approx. Scale 1" = 30'

Conceptual Typical Section for Rubblemound Breakwater

FIGURE 8